HYDROLOGY REPORT

The drainage basin characteristics for Pulpit Harbor Bridge on North Shore Road in North Haven over Mill Stream were provided by the MaineDOT Environmental Office. The peak flows are calculated using the 1999 USGS full regression equations and no other flow data is available.

SUMMARY

Drainage Area	2.43	mi^2
Q1.1	22.9	ft^3/s
Q10	88.4	ft^3/s
Q25	112.8	ft^3/s
Q50	131.8	ft^3/s
Q100	152.5	ft^3/s
Q500	202.4	ft^3/s

At Pulpit Harbor Bridge, Mill Stream is a tidal estuary, and tidal elevations were calculated using Portland as a reference station and Pulpit Harbor as a subordinate station. See Appendix E for calculations. Data is from NOAA CO-OPS and can be accessed at http://tidesandcurrents.noaa.gov/

SUMMARY

Mean Lower Low Water (MLLW)	-5.64	ft
Mean Low Water (MLW)	-5.27	ft
Mean Tide Level (MTL)	-0.32	ft
Mean High Water (MHW)	4.57	ft
Mean Higher High Water (MHHW)	5.00	ft
2013 Predicted High Tide	7.28	ft

The FEMA 2014 Preliminary Flood Insurance Study for Knox County, Maine gives a 1% annual chance flood stillwater elevation of 9.1 ft and a maximum wave crest of 12 ft for transect 57, which bisects Pulpit Harbor. The FEMA 2014 Preliminary Flood Insurance Rate Map gives a base flood elevation for Pulpit Harbor and Mill Stream of 12 ft (Panels 0217, 0219, 0236). While they are preliminary, these elevations are higher than those given in the 1991 FIS and FIRM for North Haven, so using them is conservative.

Reported by: Joshua Hasbrouck
Date: November 12, 2014

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

HYDRAULIC REPORT

The existing bridge and proposed structures were analyzed using HEC-RAS Version 4.1.0 Jan 2010, developed by the U.S. Army Corps of Engineers Hydraulic Engineering Center. The HEC-RAS model for this project represents a 308 ft long section of Mill Stream. Four stream cross-sections were created from the Department's survey data using Microstation InRoads and upstream and downstream bridge sections were created in HEC-RAS using dimensions from the survey data. The same cross-sections are used for all the existing and proposed structure hydraulic models. The following assumptions were made:

- Steady flow
- Manning's n = 0.030 for the main channel (Clean, straight, full, no rifts or deep pools)
- Manning's n = 0.050 for overbanks (Cobbles with large boulders)
- Upstream boundary condition of normal depth with a stream slope of 0.0016

The bridge is built in an area with steep, high banks with frequent ledge outcroppings, so flow will be contained even at high flows and there is no extended floodplain. The main channel is defined in the model as the width of the low tide flow, and the overbanks are the channel banks. This is not the standard definition used in HEC-RAS, but it simplifies adjusting Manning's n to account for the rougher surface on the banks and should not affect the results since the true overbanks are very high at this side and above all flows. The existing model was initially checked for all peak riverine flows with a downstream boundary condition at both MLW and MHW. For all flows, there was no change in water elevation at the bridge for MHW, and the difference between Q1.1 and Q500 water elevation for MLW was approximately 0.6 ft, so the estuary is overwhelmingly tidal controlled and detailed results of peak flow analysis will not be included here. The hydraulic models were checked for four tidal flow cases following guidance in the BDG and HEC-25:

1. Q50 flow with a downstream boundary condition at MHW
2. Q50 flow with a downstream boundary condition at MLW
3. Q1.1 flow + MHW tidal prism flow with a downstream boundary condition at MTL
4. Q1.1 flow + prism flow based on stillwater flood (also called storm surge) elevation with downstream boundary condition at MTL

The first two cases are specified in the BDG for steady flow analysis of tidal areas. Case 3 is used as a rough check of maximum tidal flow velocities at mid tide, but minimal data is available to calibrate it so it will be used qualitatively only. Case 4 checks an extreme worse case of velocities assuming water recedes from a 1% probability storm surge at the same time the tide is going out. This has a less than 1% chance of occurrence and is used as a qualitative comparison of stream velocity similar in improbability to the Q500 riverine flow usually used as an upper bound for scour.

EXISTING BRIDGE

The existing 5-span bridge was modeled in HEC-RAS using solid full height piers and sloped abutments. The existing superstructure varies in depth, but is simplified as the deepest beam depth across all the spans. Since this is above the 1% FEMA flood elevations, it will not affect the results. Water level for all cases is controlled by the ocean water level.

The low chord of the structure is at approximately 13 ft elevation, giving 8 ft of clearance at MHHW. The standard given in the BDG is a minimum of 2 ft freeboard for Q10 elevation including wave height. Pulpit Harbor Bridge is clearly well above this. The BDG is a bare minimum, however, and the clearance should be greater whenever possible. For this case, the clearance is compared to the 1% flood elevations calculated by FEMA. The maximum wave crest height in the Preliminary 2014 FIS is 12 ft. This leaves only 1 ft of freeboard; however, the wave heights are calculated for a path traveling directly into Pulpit Harbor and Mill Stream enters the harbor at right angles, so the wave height at Pulpit Harbor Bridge is likely slightly lower. It is unclear from the FIS report whether sea level rise was included in the analysis or not. Assuming it was not, the lower chord on the existing bridge could be at the maximum wave height during a storm surge in the next 100 years.

Velocities at the bridge during mid-tide when there is the greatest flow were calculated using the Tidal Prism method as outlined in HEC-25. This results in mid-tide velocities of roughly 2.5 ft/s, which matches the field measured velocities. Field measurements were taken both in the main channel by timing sticks passing under the bridge and in a shallow side channel using a handheld water velocity meter. Both methods gave a result of 2.2-2.6 ft/s during tide change. Measurements at different depths using the velocity meter indicated that the highest velocity was at around one third of the total depth, measured from the surface. This is not an exact calibration, but indicates that the hydraulic model is essentially correct. No calibration is available for the extreme case of flood water receding along with the tide, but based on the results for the normal tidal prism, the velocities for the extreme case are assumed to be similar to actual flood values.

The existing bridge substructures have been in place since before 1930 when a survey was done and in their current configuration since 1956 when the existing superstructure was constructed. As a result, any scour due to normal tidal or stream action should have already occurred and there is no scour risk for the existing bridge except in an extreme event such as Case 4. Rough calculations using a visually estimated D50 confirm this.

REMOVING ALL PIERS AND REUSING EXISTING ABUTMENTS

The third replacement option removes all the piers from the bridge and reuses the existing abutments. The approximate span length is 127'.

Calculated differences in elevations with the existing bridge model are less than 1" and are not significant. The qualitative check of velocities using the tidal prism indicates that they are, as expected, less than the existing bridge and possibly as low as 1.3 ft/s. Since this option includes a significant increase in flow area and decrease in velocities, scour in the channel is not expected to be an issue for this option. At least one of the abutments shows some damage and scour countermeasures at the base of the abutments may be needed as part of their rehabilitation.

CONCLUSIONS

All options research (some are not shown here) are hydraulically feasible and should not require any additional hydraulic analysis. The best option hydraulically is removing the piers and reusing the existing abutments, since it has the largest opening area and lowest velocities, but the other options considered are still a slight improvement over the existing bridge. Since the existing bridge has survived for almost 60 years in its current configuration without significant hydraulic issues, these options should be similar in lifespan and are viable alternatives.

Based on the low chord elevation of 13 ft and the 1% probability flood maximum wave height of 12 ft, there is a possibility that flooding could reach the underside of the bridge at some point. Therefore, a raise in profile would be preferable and a composite or concrete structure would provide the longest life. The final structure type determination will be based on other factors than hydraulics, such as cost and constructability.

SUMMARY

		Existing Structure	Proposed Structure
		5 Span Granite Substructure	127' Single Span
Total Area of Waterway Opening	ft^2	1443	2308
Headwater elevation @ Q_{50} MHW	ft*	4.6	4.6
Headwater elevation @ Q_{50} MLW	ft*	-4.7	-4.7
Headwater elevation @ Normal Prism	ft*	-0.3	-0.3
Headwater elevation @ Flood Prism	ft*	-0.1	-0.2
Freeboard @ Q_{50} MHW	ft	8	8
High Tide (2014) Elevation 7.28 ft			
1% Probability Storm Flood Elevation 12 ft			
Velocity Difference from Existing:			
Q50 MHW	ft/s†	0.0	-0.1
Q50 MLW	ft/s†	0.0	-0.7
Tidal Prism -- Normal Tides	ft/s†	0.0	-1.2
Tidal Prism -- 1% Storm Flood	ft/s†	0.0	-2.0

* Assumed accuracy of results is to nearest 0.5 ft.

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Reported by: Joshua Hasbrouck
Date: June 1, 2016

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.